

Chapter 9

Connections

Connections are one of the most critical sections of any bridge (existing or under construction). All the detailed planning, layout, and construction of a new bridge can go to waste if connections of key joints are made with inadequate materials that fail under the varying stresses in the bridge. The analytical classification and design procedures assume good connections of appropriate connectors. This chapter focuses on the most common types and selection criteria of connections.

TIMBER CONNECTIONS

9-1. Many timbers are connected by nails or spikes. Nails or spikes are normally used in handrails and knee braces to fasten timber decking to the bridge stringers. For critical connections where a high-strength fastener is required, substitute bolts for nails. Connections that require bolts also include stringer splicing and connection of the substructure members.

NAILS OR SPIKES

9-2. To create an adequate connection using nails or spikes, drive the nail or spike into the timber member to a depth no less than one-half (preferably two-thirds) the length of the fastener (*Figure 9-1*). The engineer in charge of the project must use judgment and experience to determine the quantity of the fasteners required to make the connection. Drive enough nails or spikes (in suitable patterns) to provide a rigid and durable connection. *Table 9-1, page 9-2*, gives data for the various sizes of standard nails and spikes used in bridge construction.

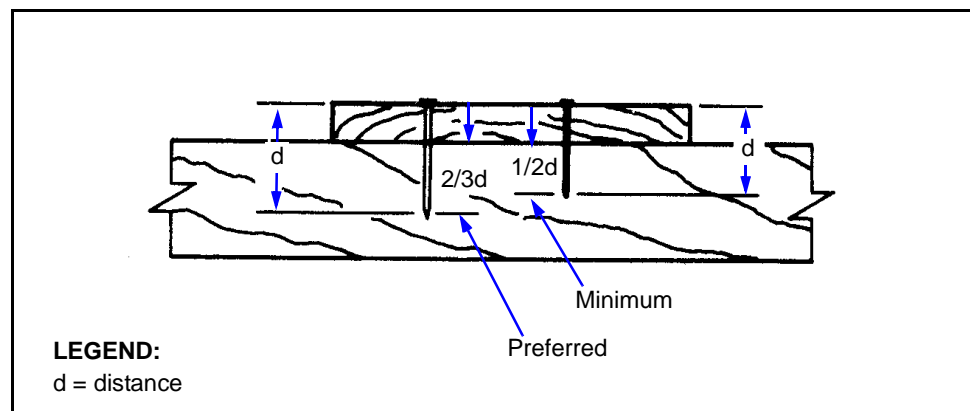


Figure 9-1. Fastener Penetration in Timber Connections

Table 9-1. Data for Nails and Spikes

Type	Size	Length (in)	Gauge	Diameter (in)
Nails	10d	3	9	0.1483
	12d	3 1/4	9	0.1483
	16d	3 1/2	8	0.1620
	20d	4	6	0.1920
	30d	4 1/2	5	0.2070
	40d	5	4	0.2253
	50d	5 1/2	3	0.2440
	60d	6	2	0.2625
Spikes	—	7	5/16	5/16
	—	8	3/8	3/8
	—	9	3/8	3/8
	—	10	3/8	3/8
	—	12	3/8	3/8

LAG SCREWS

9-3. Use lag screws instead of nails when the possibility of tension failure exists in timber members. Compute the allowable load on a lag screw by using the *Hankinson* formula as follows:

$$P_a = \frac{(P)PY}{P \sin^2 \theta + PY \cos^2 \theta} \quad (9-1)$$

where—

P_a = allowable load at an angle to the grain

P = allowable load parallel to the grain (Table C-1, pages C-3 through C-6)

PY = allowable load perpendicular to the grain (Table C-1)

θ = angle between the load and the grain

9-4. Allowable loads can vary greatly because of the variance in timber grade and type. For this reason, do not consider the strength of lag screws when designing connections. Consider only the spacing and edge distance. Always use washers under the lag-screw head.

9-5. Space lag screws apart at a minimum of four times the diameter of the screw, measured in the direction of the tensile or compressive load. For compression members, the end distance should be at least four diameters. For tension members, spacing should be at least seven diameters. For loading perpendicular to the grain, the loaded edge distance should be at least four times the diameter of the screw. When the load is parallel to the grain, one

and one-half diameters is a sufficient edge distance. Measure all distances from the center of the lag screws (*Figure 9-2*).

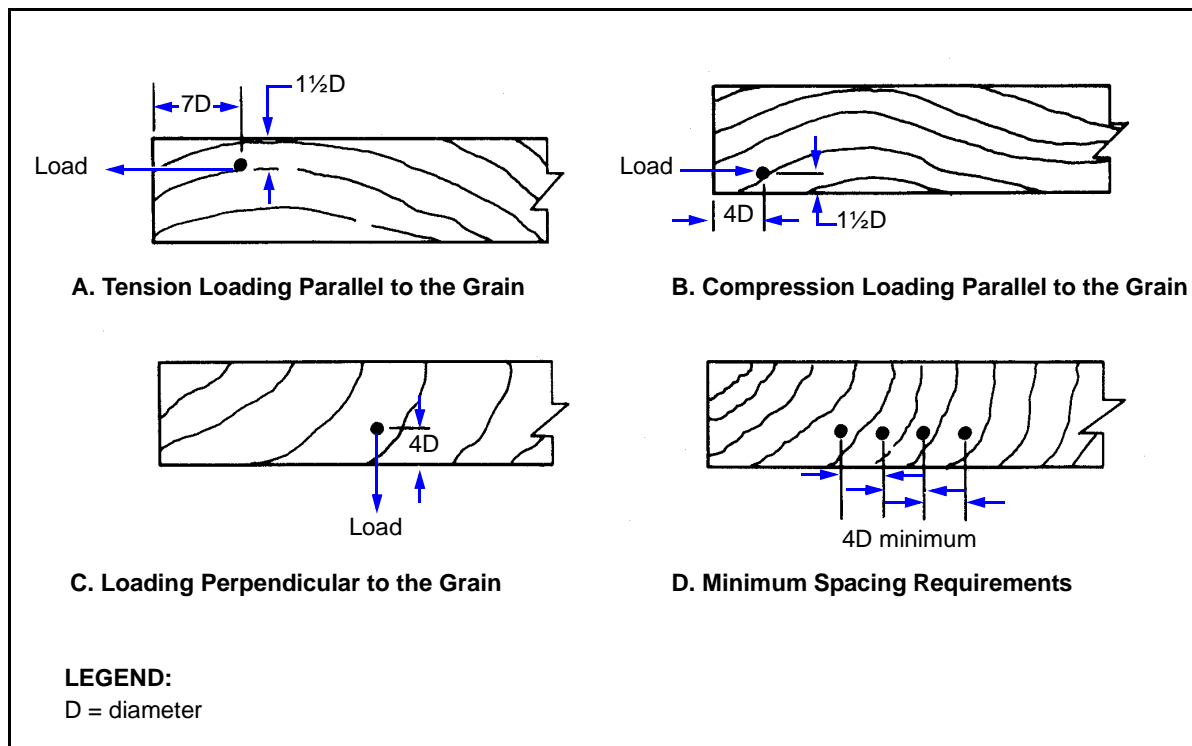


Figure 9-2. Edge Distance and Spacing Requirements

BOLTS

9-6. Critical timber connections require bolts. Bolts provide a strong, efficient, and economical method of fastening wood members together. Bolts are available in a variety of sizes and can be used with all sizes of timber. Use bolted timber connections to splice stringers, connect substructure members, and attach deck and curb material. Also use them for fastening wood to wood and for fastening steel plates to wood members. Always use washers under the bolt head and bolt nut.

Maximum Allowable Loads

9-7. *Table 9-2, page 9-4*, shows the maximum allowable loads (normal loads) for commonly used bolts. These tabulated loads are for bolted joints in lumber which is seasoned to a moisture content nearly equal to that it will attain in service. The loads are for bolted joints used under continuously dry conditions, as in most covered structures. For joints that will be exposed to weather or will always be wet, reduce the tabulated loads to 75 and 67 percent, respectively. Place a washer or metal plate between the wood and the bolt head and between the wood and the nut. Use the loads given in *Table 9-2* when each of the side members of wood is half the thickness of the main (enclosed) member (*Figure 9-3A, page 9-5*).

Table 9-2. Allowable Bolt Loads (kips per bolt)

Bolt Diameter (in)	Portion (Length) of Bolt in Member (in)						
	1 5/8	2	4	6	8	10	12
1/2	1.01	1.18	1.29	—	—	—	—
	0.48	0.59	1.04	—	—	—	—
5/8	1.29	1.56	2.01	2.01	2.01	—	—
	0.54	0.67	1.33	1.42	1.26	—	—
3/4	1.55	1.91	2.89	2.89	2.89	—	—
	0.60	0.74	1.49	1.97	1.84	—	—
7/8	1.81	2.23	3.83	3.94	3.94	3.94	—
	0.67	0.82	1.64	2.41	2.47	2.29	—
1	2.07	2.55	4.72	5.14	5.14	5.14	5.14
	0.73	0.89	1.79	2.68	3.15	3.00	2.79
1 1/4	2.59	3.18	6.22	7.85	8.04	8.04	8.04
	0.84	1.03	2.06	3.10	4.17	4.61	4.45
1 1/2	3.10	3.82	7.56	10.60	11.60	11.60	11.60
	0.99	1.21	2.42	3.63	4.78	5.44	5.44
1 3/4	3.62	4.45	8.91	12.87	15.28	15.90	15.90
	1.10	1.36	2.72	4.09	5.44	6.63	7.21
2	4.14	5.10	10.20	15.12	18.80	20.40	20.50
	1.23	1.51	3.03	4.54	6.05	7.56	8.77
NOTE: The top number is the load parallel to the grain. The bottom number is the load perpendicular to the grain.							

9-8. If the side members are more than half the thickness of the main member, do not increase the tabulated load (*Figure 9-3B*). When the side members are less than half the thickness of the main member, the tabulated load for a main member is twice the thickness of the thinnest side member. For example, with 3-inch side members and an 8-inch main (center) member, the tabulated loads for an 8-inch main member apply (*Figure 9-3C*).

9-9. When the joint consists of two members of equal thickness (the bolt is in single shear), use one-half the tabulated load for a piece twice the thickness of one of the members (*Figure 9-4A*). When members of a two-member joint are of unequal thickness, use one-half the tabulated load for a piece twice the thickness of the thinner member (*Figure 9-4B*). When using steel plates for side members, increase the tabulated loads for a parallel-to-grain loading by 25 percent. Do not increase the tabulated loads for perpendicular-to-grain loads (*Figure 9-5, page 9-6*).

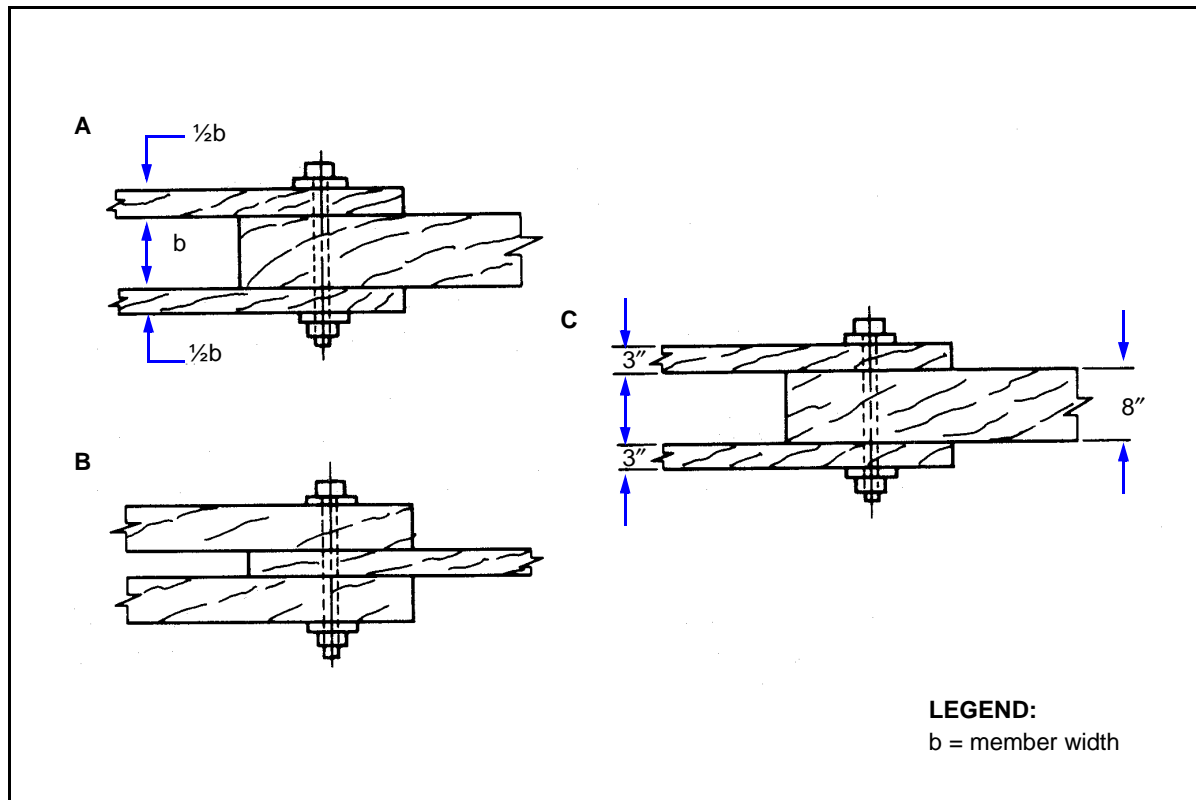


Figure 9-3. Bolted Double-Shear Connections

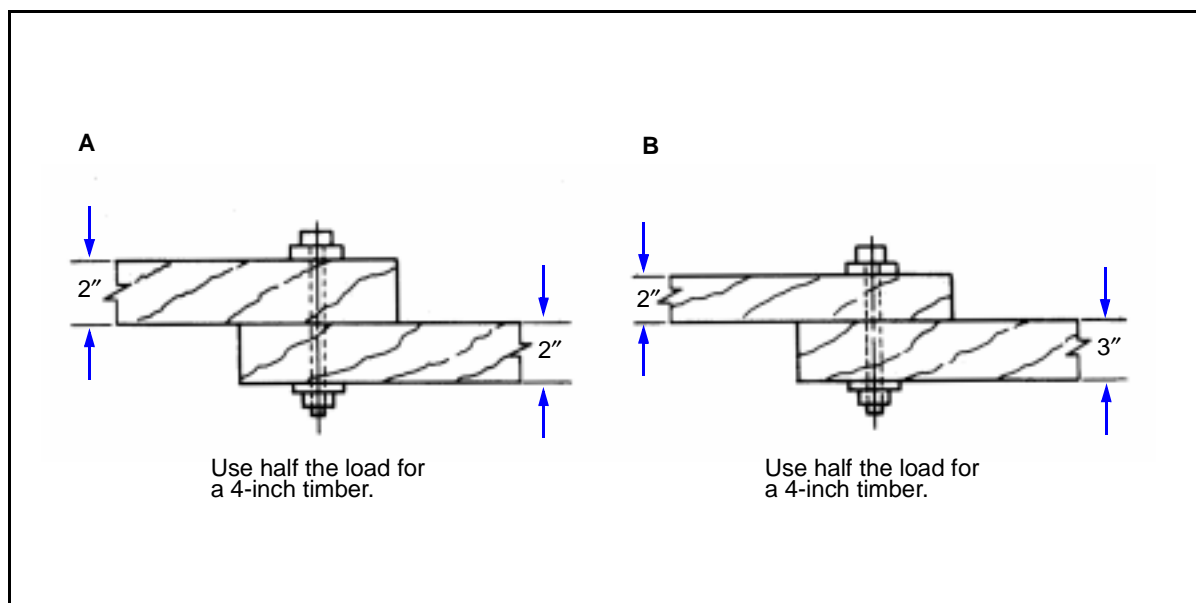


Figure 9-4. Bolted Single-Shear Connections

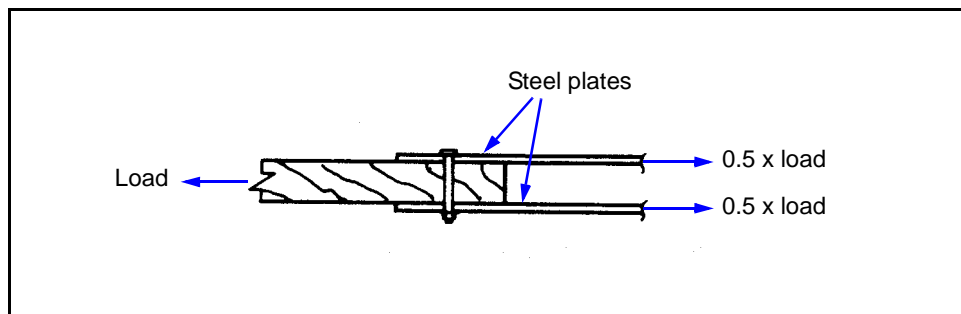


Figure 9-5. Steel-to-Timber Bolted Connections

Allowable Pressure

9-10. The direction of bolt pressure on the grain of the wood must be considered when determining the allowable pressure. *Table 9-2, page 9-4*, gives the allowable pressure for bolts placed parallel and perpendicular to the grain. *Figure 9-6* shows that the bolt pressure is parallel to the grain of the inclined member. In the horizontal member, however, the load is neither parallel nor perpendicular to the grain. Use *equation 9-1* to find the allowable bolt load on the horizontal member.

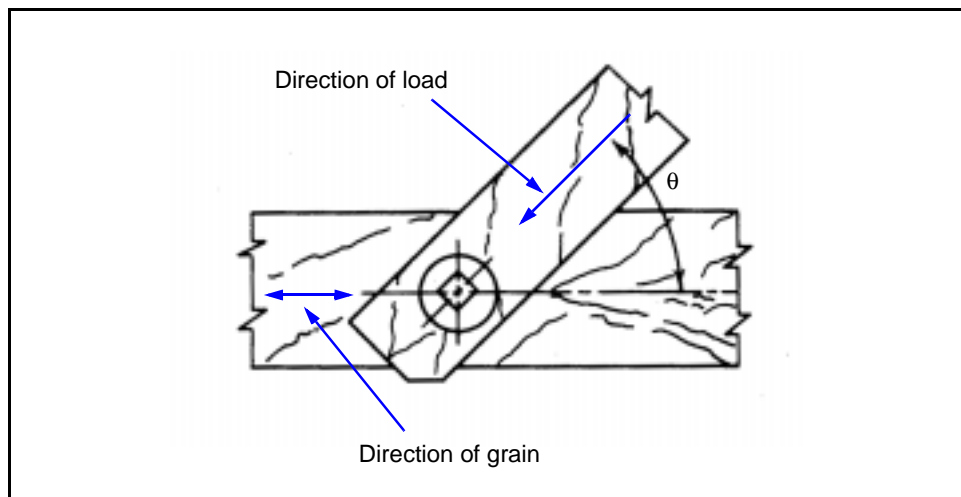


Figure 9-6. Timber Connections with the Load Applied at an Angle to the Grain

Design Criteria

9-11. In bolted connections, the allowable load that the joint supports must not exceed the allowable bolt load of one bolt multiplied by the number of bolts used. Criteria include the following:

- The net cross-sectional area of the member (measured at a right angle to the direction of the load) is the gross cross-sectional area of the member minus the area of the bolt holes in the section.

- The allowable tensile joint load for any bolted joint load must not exceed the net cross-sectional area multiplied by the allowable tensile unit stress of the lumber.
- The net area for softwoods at the critical section for parallel-to-grain loading must be at least 80 percent of the total area in bearing under all bolts in the member. For hardwoods, the net area must be at least 100 percent of the total area.

9-12. Data on the species and sizes of the members to be joined is required for designing a bolted joint. Assume a bolt size and then find the allowable load for one bolt. To compute the required number of bolts, divide the total load by the allowable load per bolt. The values given in *Table 9-2, page 9-4*, are based on the maximum allowable stresses for average structural timbers and are 1,800 psi parallel to the grain and 500 psi perpendicular to the grain. If the timber to be used has different allowable stresses, increase or decrease the tabulated value of the load, as needed. Do this by multiplying the tabular stress by the ratio of allowable stress for the chosen timber to 1.8 ksi (parallel to the grain) or 0.5 ksi (perpendicular to the grain).

Spacing

9-13. Bolt spacing is critical. Base the bolt spacing and the minimum required edge distance on the criteria described in *Figures 9-2, page 9-3*, and *Figure 9-3, page 9-5*.

Washers

9-14. Add washers to both sides of the bolted timber connections to prevent bearing failure in the timber. Use wrought-iron or steel-plate round washers. *Table 9-3* shows the washer requirements for bolted connections.

Table 9-3. Washer Requirements for Bolted Connections

Bolt Diameter (in)	Hole Size (in)*	Washer Diameter (in)	Washers per Pound (approx)
1/2	9/16	1 3/8	27.0
5/8	11/16	1 3/4	13.0
3/4	13/16	2	10.0
7/8	15/16	2 1/4	8.6
1	1 1/16	2 1/2	6.2
1 1/4	1 3/8	3	4.0
1 1/2	1 5/8	3 1/2	2.5
1 3/4	1 7/8	4	2.2
2	2 1/8	4 1/2	1.7
*The washer-hole size is 1/16 inch larger than the diameter of the bolt for all bolts up to and including 1 inch. For larger bolts, use a washer-hole size that is 1/8 inch larger than the diameter of the bolt.			

FLOOR CLIPS

9-15. Use floor clips to connect a laminated wooden deck to steel stringers. Position the floor clips with the anchor end under the flange and then nail them to the lamination at 16- to 18-inch intervals. Install floor clips in pairs, one on each side of the stringer (*Figure 9-7*). If floor clips are not available or a laminated deck is not used, use one of the expedient methods shown in *Figure 9-8*. Fasten the deck to the stringers before placing the stringers. The objective is to prevent lateral movement of the decking.

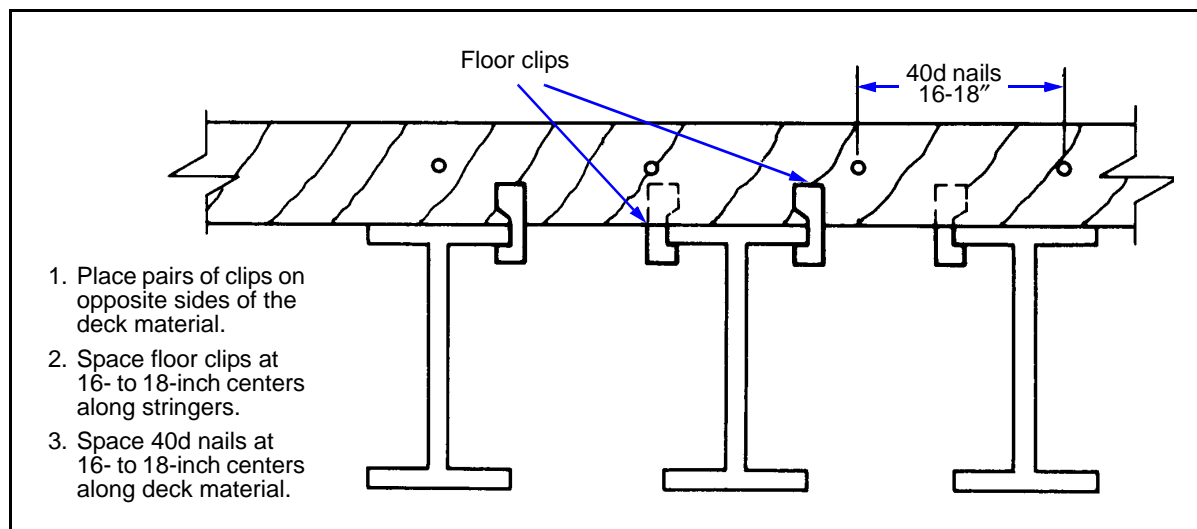


Figure 9-7. Deck Connections for Laminated Decking

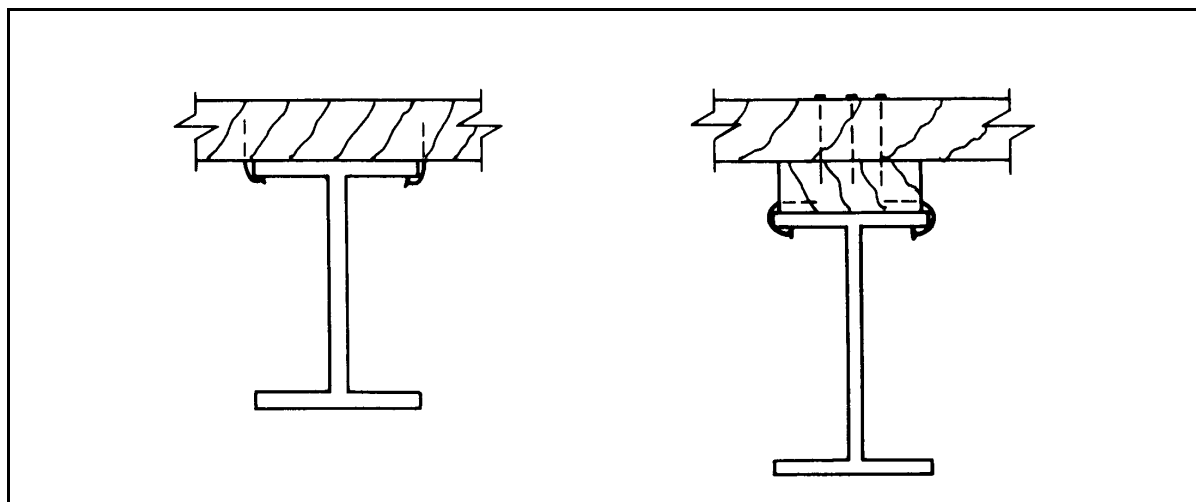


Figure 9-8. Expedient Deck Connections

OTHER CONNECTORS

9-16. Shear plates, spike grids, clamping plates, and split and toothed-ring connectors are specially designed devices for making timber-to-timber

connections. However, these connectors are intended for light-frame construction, which limits the application of these connectors in military-bridge construction.

STEEL CONNECTIONS

9-17. In the AFCS, preengineered structures come complete with bolts and prepunched, matched holes. Such structures make construction easier and quicker.

BOLTS

9-18. Conditions may arise that call for the design and fabrication of bolted steel connections (including high-strength bolts). The design process for bolted steel connections involves determining spacing and evaluating connection strengths and possible failures.

Bolt Spacing

9-19. The minimum center-to-center bolt spacing is three times the diameter of the bolt. *Figure 9-9* shows the typical bolt spacing for steel angles. The minimum distance from the edge of a member to the center of the nearest bolt is the edge distance. *Table 9-4, page 9-10*, lists edge distances for some of the more common bolt sizes.

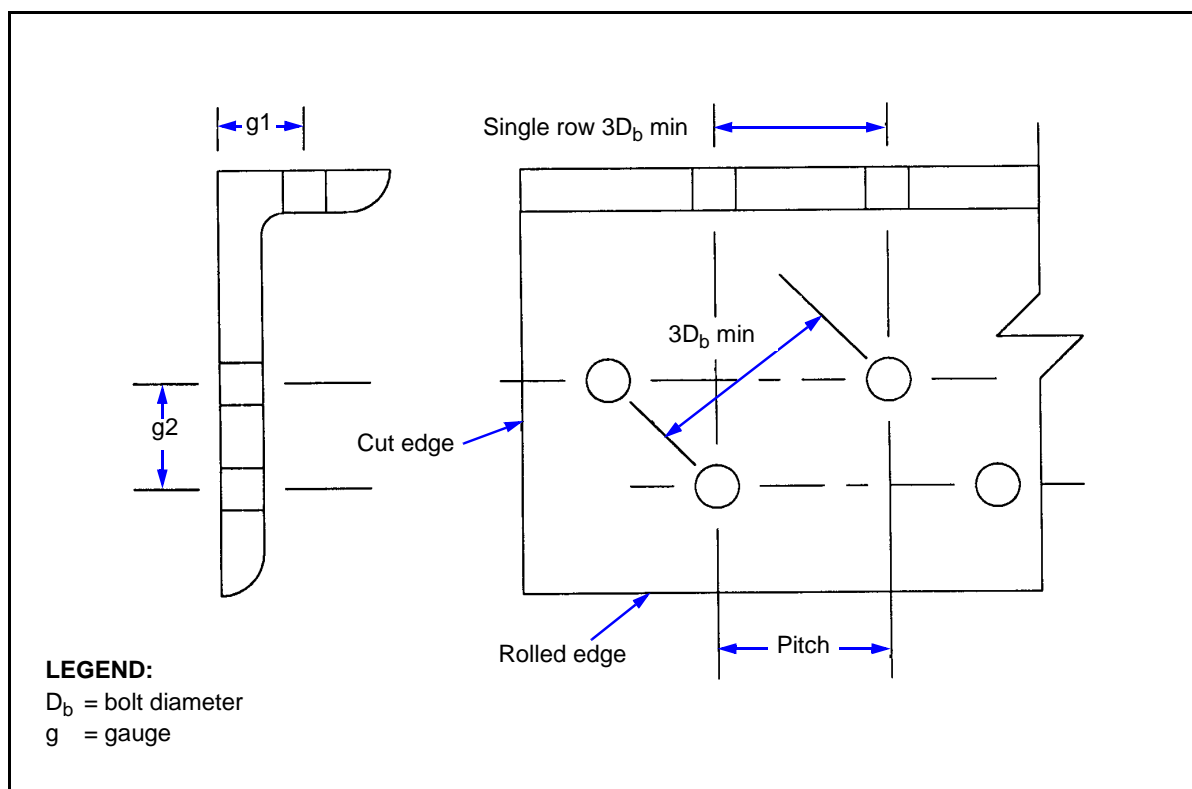


Figure 9-9. Bolt Spacing

Table 9-4. Edge Distances

Gauge	Leg Length (in)												
	8	7	6	5	4	3 1/2	3	2 1/2	2	1 3/4	1 1/2	1 1/4	1
g	4 1/2	4	3 1/2	3	2 1/2	2	1 3/4	1 3/8	1 1/8	1	7/8	3/4	5/8
g1	3	2 1/2	2 1/4	2	*	*	*	*	*	*	*	*	*
g2	3	3	2 1/2	1 3/4	*	*	*	*	*	*	*	*	*
*Single row only.													

Failure Modes

9-20. The strength of a bolted connection is governed by the smallest of the following:

- Shear strength of the bolts.
- Bearing strength of the bolted members.
- Tensile strength of the bolted members at the weakest section.

To determine the strength of a connection, first evaluate the failure modes (shear, bearing, tension, or a combination of all three).

9-21. **Shear.** The most common type of bolt failure is shear. Shear occurs when the applied forces exceed the bolts' allowable shear capacity. In dealing with shear, first determine whether the connection is a single- or double-shear connection. The allowable shear capacity for a double-shear connection is twice that of a single-shear connection. *Figure 9-10* illustrates the typical shear failure for both connections. Double shear is preferred where the eccentricity of the applied loads might induce serious damage. Determine the shear capacity of a bolt by using *Table 9-5* or the following equation:

$$v = AF_v \quad (9-2)$$

where—

v = shear capacity of one bolt, in kips

A = area of one bolt (single shear), in square inches (double the area for double shear)

F_v = allowable shear stress of the steel, in ksi (0.49F_y)

9-22. **Bearing.** Bearing failure is the tendency of a bolt to enlarge its hole. Determine the bearing capacity of a bolt by using *Table 9-5* or the following equation:

$$P_B = DF_B t \quad (9-3)$$

where—

P_B = bearing capacity per bolt, in kips

D = bolt diameter, in inches

F_B = allowable bearing stress of the steel, in ksi (Table 3-6)

t = thickness of the thinner piece of steel, in inches

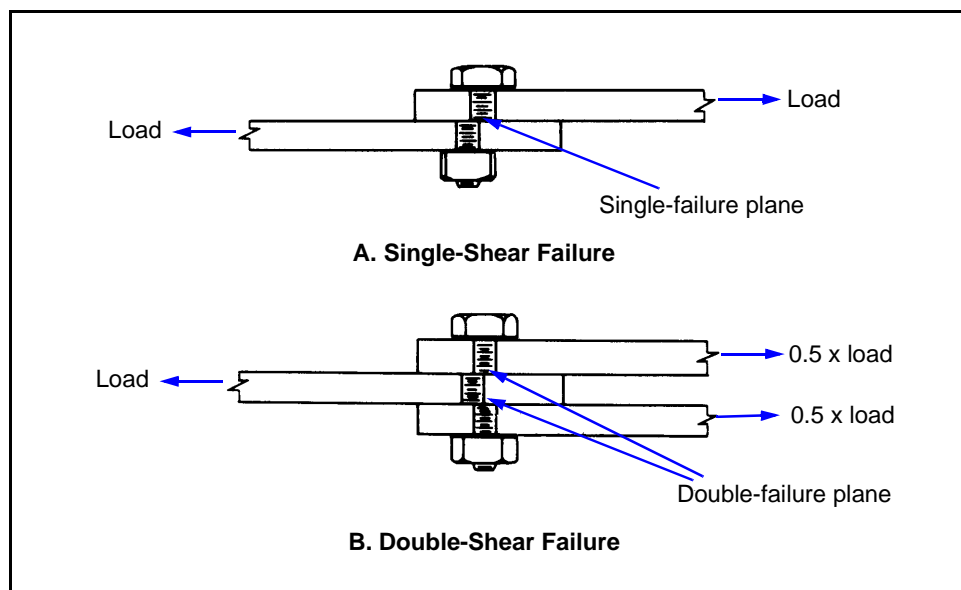


Figure 9-10. Shear Failure in Bolted Connections

Table 9-5. Bolt Shear and Bearing Capacities

Bearing Capacity per Bolt (kips)						
Material Thickness (in)	Bolt Diameter (in)					
	5/8	3/4	7/8	1	1 1/8	1 1/4
1/8	3.52	4.22	4.92	5.63	6.33	7.03
3/16	5.27	6.33	7.38	8.44	9.49	10.55
1/4	7.03	8.44	9.84	11.25	12.66	14.06
5/16	8.79	10.55	12.30	14.06	15.82	17.58
3/8	10.59	12.66	14.77	16.88	18.98	21.09
7/16	12.30	14.77	17.23	19.69	22.15	24.61
1/2	14.06	16.88	19.69	22.50	25.31	28.13
9/16	15.82	18.98	22.15	25.31	28.48	31.64
5/8	17.58	21.09	24.61	28.13	31.64	35.16
11/16	19.34	23.20	27.07	30.94	34.80	38.67
3/4	21.09	25.31	29.53	33.75	37.97	42.19
13/16	22.85	27.42	31.99	36.56	41.13	45.70
7/8	24.61	29.53	34.45	39.38	44.30	49.22
15/16	26.37	31.64	36.91	42.19	47.46	52.73
1	28.13	33.75	39.38	45.00	50.63	56.25
Shear per Bolt (kips)						
Type	Bolt Diameter (in)					
	5/8	3/4	7/8	1	1 1/8	1 1/4
Single	3.68	5.30	7.22	9.42	11.93	14.73
Double	7.36	10.60	14.44	18.84	23.86	29.46

9-23. **Tensile.** Compute the tensile strength as follows:

$$T = AF_t \quad (9-4)$$

where—

T = tensile strength of the member, in kips

A = net area of the member, in square inches. (The net area of the member is its gross cross-sectional area minus the total of the area removed for bolt holes at a given section. When determining the net area, use a diameter for the bolt hole equal to 1/8 inch greater than the bolt diameter. If the computed net area is greater than or equal to 85 percent of the gross area $[Ag]$, use $0.85Ag$ instead of the computed net area.)

F_t = allowable tension of the member, in ksi (29 ksi for steel)

9-24. **Combination.** After determining the shear and bearing capacities for a single bolt, determine the strength of the connection by examining the various modes of failure. A connection may fail by simultaneous shear, bearing, or tensile failures in any combination among the various rows of bolts.

WELDS

9-25. When equipment and trained personnel are available, use welded instead of bolted connections. Minor connections for which stress is not computed (such as nailing clips) may be welded without a formal design. However, weld major load-carrying connections only after careful design. These connections require trained and supervised welders. If in doubt about the quality of welding equipment or the welders' level of training, use bolted connections.

9-26. Fabricating welded connections is critical. A weld design is wasted if not followed correctly. Welded connections require designs suited to bridge construction. Do not weld connections that are specifically designed to be bolted. Work is easier if the component parts are clamped together or secured by several fitting-up bolts before welding. Experienced, well-trained welders are essential, since satisfactory welded connections depend entirely on the skills of the welders. Special training and skill are also required to inspect welded connections properly, since welds may look sound but be inferior. Welds are economical and, when properly placed, are as dependable as bolted connections.

Welding Process

9-27. The two principal welding processes for structural work are electric-arc and oxyacetylene welding. In electric-arc welding, the electrical arc formed between a suitable electrode and the base metal develops the welding heat. The electrode is generally steel (use only shielded-arc electrodes). The electric-arc process is commonly used for structural welding. In oxyacetylene welding, welding heat is obtained by burning an oxygen-acetylene gas mixture discharged under pressure from a torch designed specifically for the purpose. Oxyacetylene welding is preferred for butt welds when joining two heavy pieces of metal. *Figure 9-11* shows some basic welding terminology.

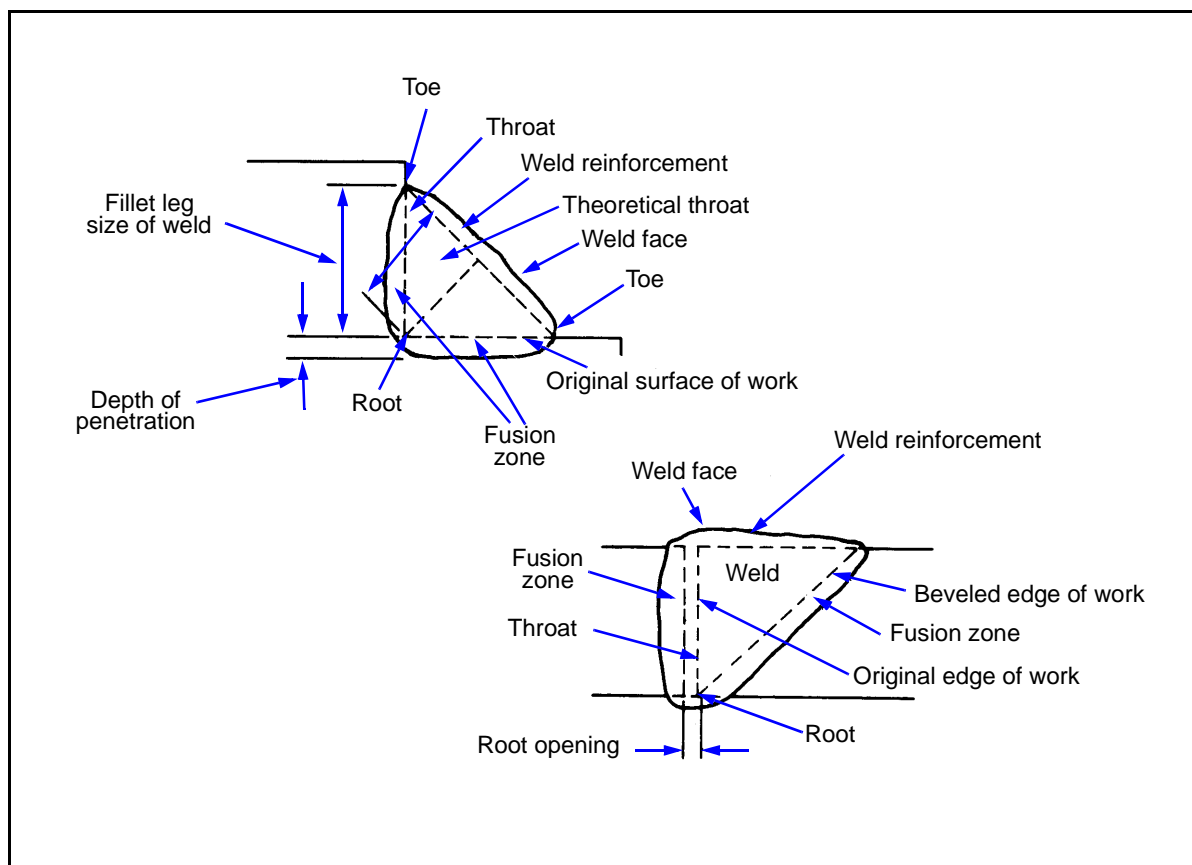


Figure 9-11. Basic Welding Terminology

9-28. When choosing a welding design, consider such factors as metal type, expansion and contraction, and post-weld inspection. Welding creates considerable heat. As a result, dangerous contractions occur during cooling that create internal residual stresses, possible deformities, and loss of strength. Design symmetrical welds to counter some of these stresses but do not overdesign. Peening and annealing remove much of the residual stress. Annealing in the field is impractical. Therefore, always peen field welds to remove residual stresses, regardless of their sizes. After welding, the base metal is more brittle than before, depending on the rate of cooling. The more carbon a metal contains the more difficult it is to weld. Preheating members provides a satisfactory treatment. In small welds on thick members, unequal heating creates unequal contractions. Larger members heat up far less, preventing some contraction and producing residual stresses. *Figure 9-12, page 9-14, shows examples of various faulty and acceptable weld profiles.*

Weld Types

9-29. The two basic weld types are the butt and fillet. *Figure 9-13, page 9-14, shows several variations of these weld types.*

9-30. **Butt Weld.** A butt weld does not require additional splice material. The weld metal alone provides the connection strength. The maximum stress

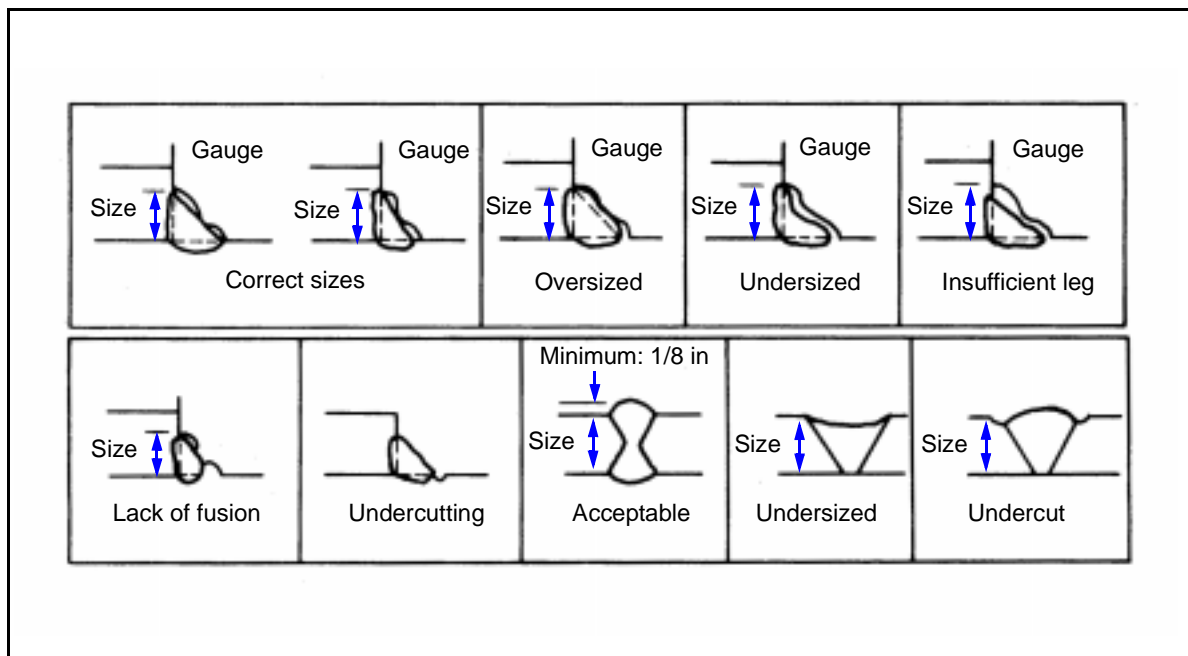


Figure 9-12. Faulty and Acceptable Weld Profiles

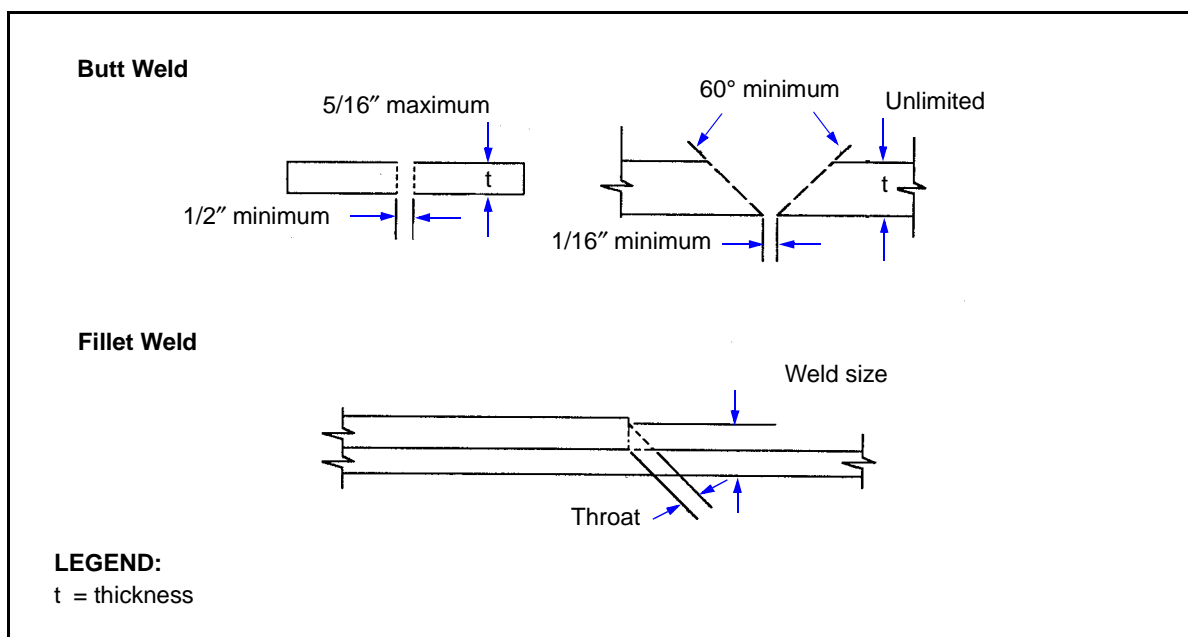


Figure 9-13. Typical Butt and Fillet Welds

permitted for butt welds is the same as that of the base metal of the parts joined. For square-ended parts where no special machining is required, the maximum thickness of the parts is 5/16 inch (*Figure 9-13*). The throat of the butt weld is the thickness of the thinner section. When parts are specially machined, the thickness of the welded part is unlimited.

9-31. **Fillet Weld.** The fillet weld is the most commonly used weld because the base metal does not require special machining. The design is simple because the shear on the throat of the weld metal is the only weld stress considered. The faces of fillet welds are normally oriented 45 degrees to the plate surfaces. Where possible, avoid weld sizes larger than 5/16 inch. These sizes require more than one pass, with a subsequent increase in preparation, welding, and finishing time.

Weld Strength

9-32. **Butt Weld.** Determine the allowable butt-weld strength in the same manner as for the base metal of the parts joined.

9-33. **Fillet Weld.** Measure the fillet-weld strength by determining the shear value per linear inch of weld. The allowable shearing stress of field welds depends on the type of electrodes used. Some types of electrodes are as follows:

- E60 (produces 12.6 ksi).
- E70 (produces 13.6 ksi).
- E80 (produces 15.6 ksi).
- E90 (produces 16.2 ksi).

9-34. For a 5/16-inch fillet weld, the throat thickness is the minimum dimension of the weld ($0.707 \times 5/16 = 0.221$ inch). An E60 electrode would develop 2.787 kips per linear inch (0.221×12.6). Compute the required weld length as follows:

$$L = \frac{P}{U_w} \quad (9-5)$$

where—

L = weld length, in inches

P = load the weld must transfer, in kips

U_w = weld strength, in kips per inch

9-35. The following rules apply when considering the weld size:

- Make the weld size 1/16 inch less than the thickness of the plate if the edge of the plate is flame-cut or sheared.
- Use three-fourths of the nominal edge thickness as the weld size if the edge of the plate is rounded or rolled (such as the toes of the angles and the channel flanges).
- Make the weld size the same as the thickness of the metal to be joined. Since this is not possible in all cases, ensure that the size does not exceed one and one-half times the thickness of the metal.
- Do not allow individual weld sizes to exceed one-half the plate thickness, when welds are on both sides of a plate. Small welds are difficult to lay. Larger welds (more than one pass) require cooling delays to allow slag removal before the next pass.

- Avoid welds that are larger than 5/16 inch. *Table 9-6* lists the maximum thickness of material that may be connected by the various sizes of fillet weld.

Table 9-6. Maximum Thickness for Fillet Welds

Weld Size (in)	Minimum Plate Thickness (in)	Weld Strength (lb per in)
3/16	1/2	2,470
1/4	3/4	3,290
5/16*	1 1/4	4,110
3/8	2	4,930
1/2	6	6,580
5/8	>6	8,220
*Minimum size for a single pass.		

9-36. Balanced Weld. An important consideration in the design of fillet welds is the balanced-weld concept. Always use a balanced weld to reduce the effect of moment in an asymmetric section or in an eccentrically loaded section. For a simple case, consider an angle member (asymmetric section) that is to be loaded axially (*Figure 9-14A*). *Figure 9-14B* shows a balanced end weld.

9-37. Place more weld material at the top of the angle section than at the bottom of the section. In this way, no moment is introduced into the connection. This concept proportions the weld material in both locations, so that the moment effect is held to a minimum. Compute the total length of the required weld by using *equation 9-5*. Then compute the proportionate weld lengths as follows:

$$L_b = \frac{Ld}{L_L} \quad (9-6)$$

where—

L_b = weld length at the bottom of the angle section, in inches

L = total weld length required, in inches (*equation 9-5*)

d = distance from the neutral axis of the angle section to the outer edge of the nonwelded leg, in inches (*Figure 9-14A*)

L_L = length of the loaded angle leg, in inches

9-38. The weld length required on the opposite leg becomes the total weld length required minus the weld length at the bottom of the angle section. When welding the end of the leg of the angle section to the connecting member, the corresponding weld lengths are computed as follows:

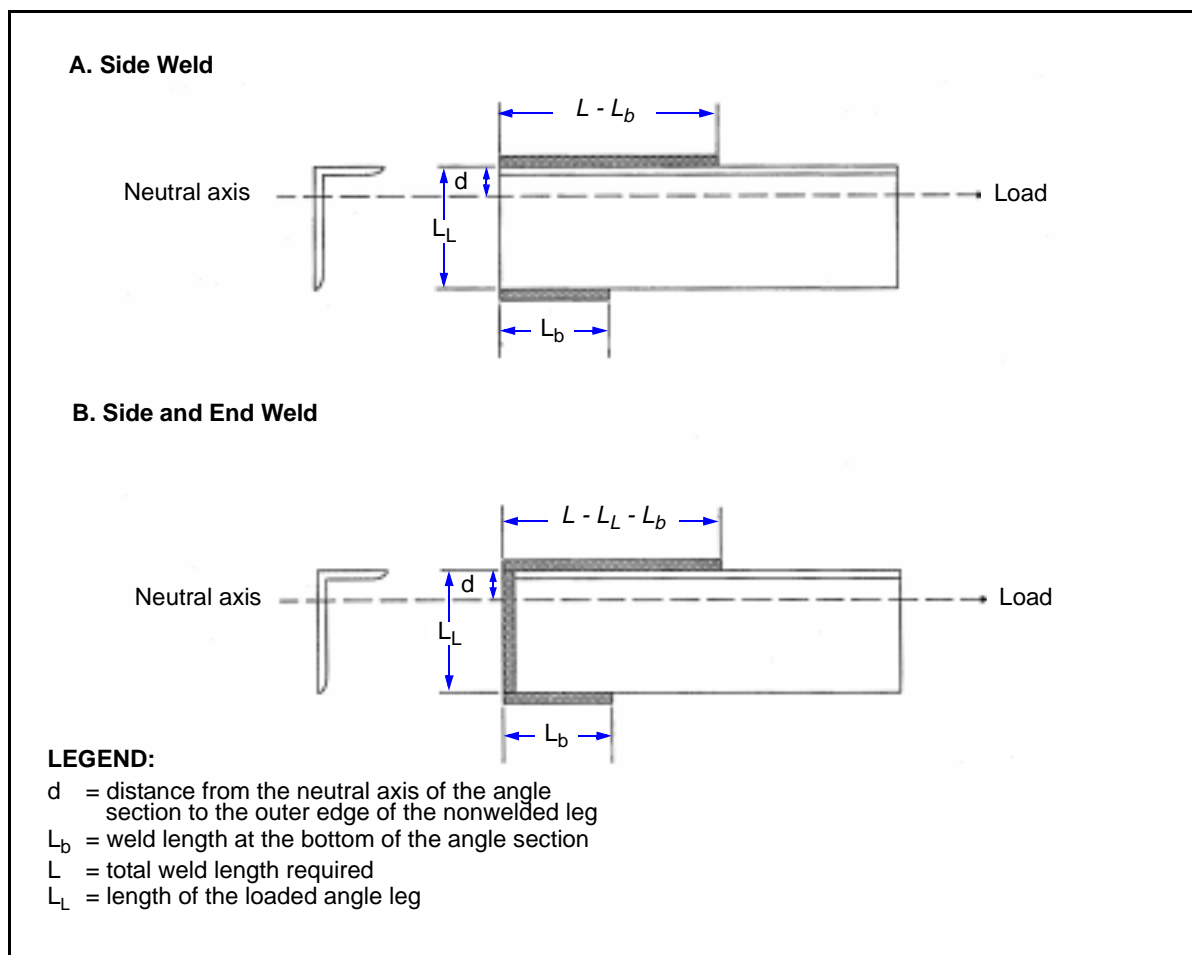


Figure 9-14. Balanced Weld Concept

$$L_b = \frac{d}{L_L} - \frac{L_L}{2} \quad (9-7)$$

where—

L_b = weld length at the bottom of the angle section, in inches

L = total weld length required, in inches (equation 9-5)

d = distance from the neutral axis of the angle section to the outer edge of the nonwelded leg, in inches (Figure 9-14A)

L_L = length of the loaded angle leg, in inches

